CHAPTER

4 Footing Foundations

General

Footing foundations transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing, in contrast to pile-supported foundations which transmit design loads into the adjacent soil mass through pile friction, end bearing, or both.

Since the load bearing capacity of most soils is quite low, about 2 to 5 Tons per Square Foot (TSF), footing areas will be large in relation to the cross section of the supported member, particularly when the supported member is a column.

Each individual footing foundation must be sized so that the maximum soil bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. In addition, footing settlement must not exceed tolerable limits established for differential and total settlement. Each footing foundation must also be structurally capable of spreading design loads laterally over the entire footing area.

Types

Footing foundations can be classified into two general categories: (1) footings that support a single structural member, frequently referred to as "spread footings", and (2) footings that support two or more structural members, referred to as "combined footings."

Although not a separate category, seismic retrofits of pre-1973 spread footings are now quite common. Designs of spread footing seismic retrofits typically include adding a top mat of rebar so that any seismic uplift force, which would produce tension in the top of the footing, can be resisted. In some cases footing dimensions are increased and/or perimeter piles added, which create a resisting couple required to provide additional restraint against rotation. Typical spread footings seismic retrofits are shown in Figures 4-1 and 4-2.

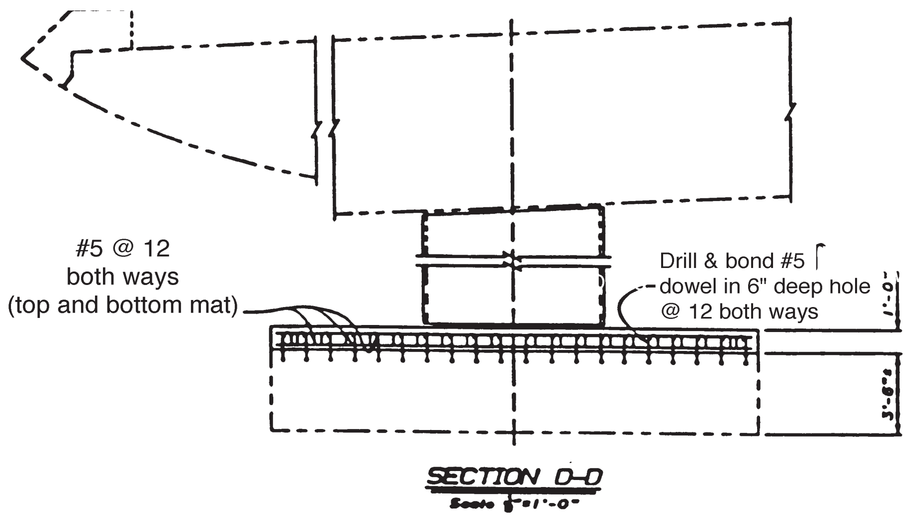


Figure 4-1: Footing Retrofit (add top mat of rebar)

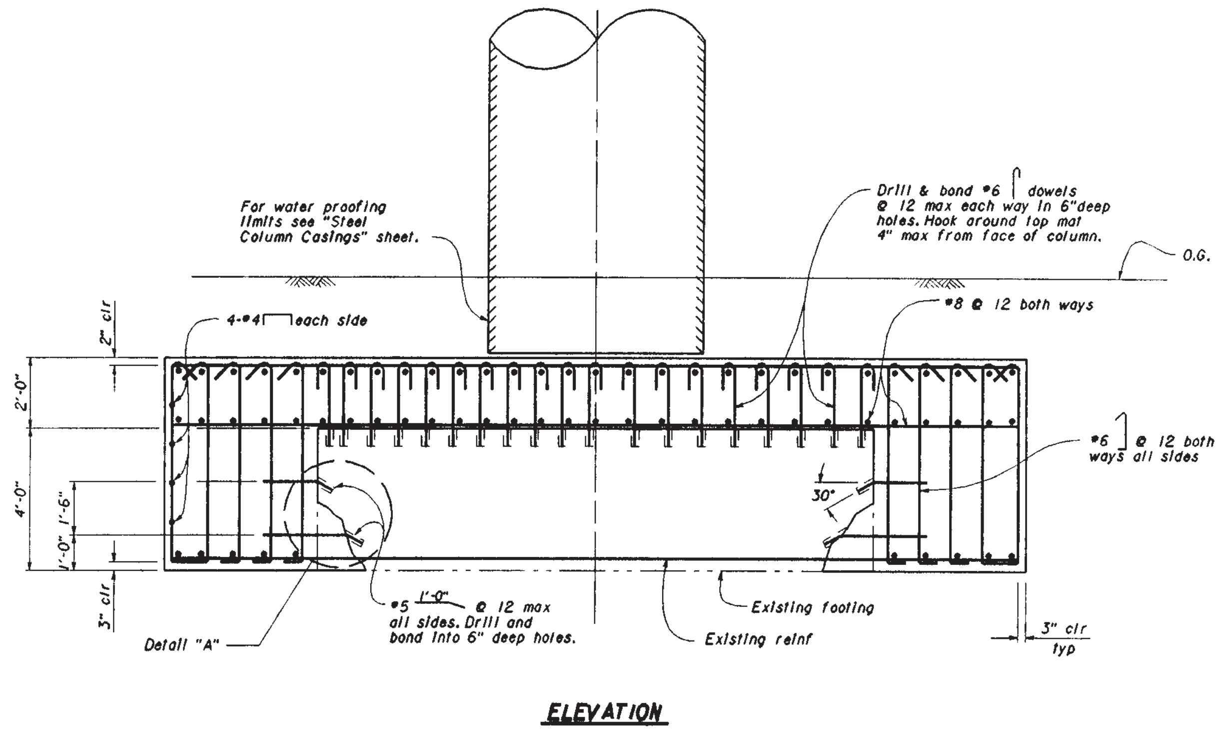


Figure 4-2: Footing Retrofit (increase footing size and add top mat of rebar)

Typically, columns are located at the center of spread footings, whereas retaining walls are eccentrically located in relation to the centerline of a continuous footing.

Combined footings are generally required when loading conditions (magnitude and location of load) are such that single column footings create undesirable engineering problems, are impractical, or uneconomical. For example, locating a column at or near a footing edge will invariably result in a soil bearing pressure that exceeds the allowable bearing capacity of the soil mass. Other potential engineering problems associated with edge-loaded footings are excessive settlement and/or footing rotation. The results of footing rotation on soil bearing pressures can be seen in Figure 4-3.

These problems can be eliminated, or at least minimized, by combining an edge-loaded footing with an adjacent single column footing. This is generally accomplished by one of two methods. In the first method, two footings are combined to form a single rectangular or trapezoidal footing. This type is referred to as a combined footing. In the other method, two spread footings (one edge-loaded) are structurally connected by a narrow concrete beam. This type is referred to as a cantilever or strap footing.

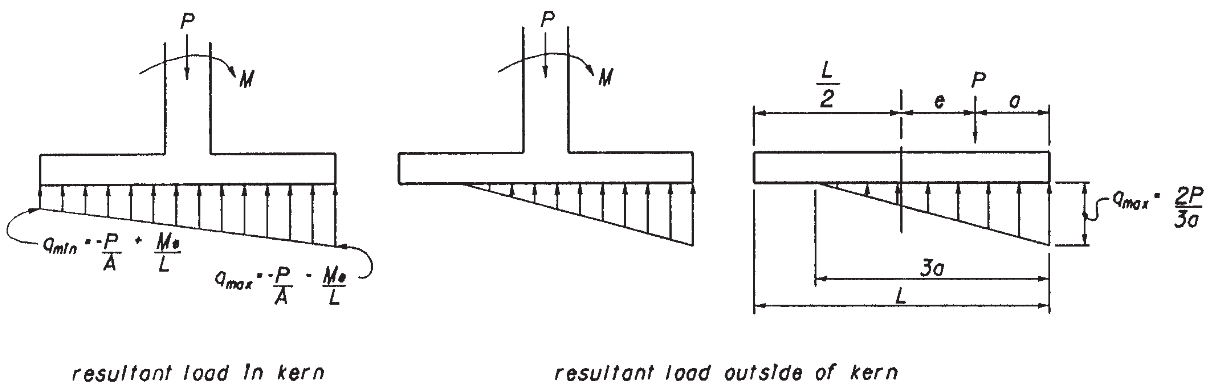


Figure 4-3: Loaded Footing with Moment

Combined footings may also be required when column spacing is such that the distance between footings is small or when columns are so numerous that footings cover most of the available foundation area. Generally, economics will determine whether these footings should be combined or remain as individual footings. A single footing supporting numerous columns and/or walls is referred to as a mat footing.

Footing foundations encountered in bridge construction almost always support a single structural member (column, pier or wall) and are invariably referred to as spread footings. Although closely spaced columns do occur in multiple column bents, they are rarely supported on a combined footing. However, recent seismic retrofit projects have incorporated designs which have attached adjacent footings together.

Bearing Capacity

The ultimate bearing capacity of a soil mass supporting a footing foundation is the maximum pressure that can be applied without causing shear failure or excessive settlement.

At present, ultimate bearing capacity solutions are based primarily on the Theory of Plasticity; that is, the soil mass is incompressible (does not deform) prior to shear failure. After failure, deformation (plastic flow) occurs with no increase in shear.

The implication of the foregoing statements is that theoretical predictions can only be applied to soils that are homogeneous and incompressible. However, most soils are neither homogeneous nor incompressible. Consequently, known theoretical solutions are used in bearing capacity analyses but are modified to provide for variations in soil characteristics. These modifications are based primarily on data obtained empirically and through small-scale testing.

Failure Modes

Shear failure of a soil mass supporting a footing foundation will occur in one of three modes: (1) general shear, (2) punching shear, or (3) local shear. The general shear failure mode can be theoretically described by the Theory of Plasticity. The other two failure modes, punching and local shear, have, as yet, no theoretical solutions.

General shear failure is shown in Figure 4-4 and can be described as follows: The soil wedge immediately beneath the footing (an active Rankine zone acting as part of the footing) pushes Zone II laterally. This horizontal displacement of Zone II causes Zone III (a passive Rankine zone) to move upward.

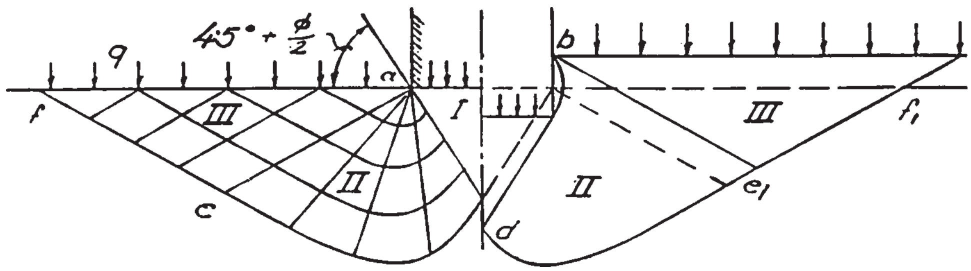


Figure 4-4: General Shear Failure Concept

General shear failure for the most part is sudden and catastrophic. Although bulging of the ground surface may be observed on both sides of the footing at a stress level below failure, failure usually occurs on one side of the footing. For example, an isolated structure may tilt substantially or completely overturn. A footing restrained from rotation by the structure will increase structure moments (stresses) and may lead to collapse or excessive settlement.

Punching shear failure (Figure 4-5) presents little, if any, ground surface evidence of failure, since the failure occurs primarily in soil compression immediately beneath the footing. This compression is accompanied by vertical movement of the footing which may or may not be observed, i.e., movement may be occurring in small increments. Footing stability (no rotation) is usually maintained throughout failure.

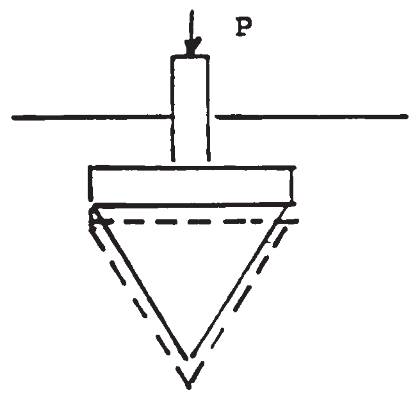


Figure 4-5: Punching Shear Failure

Local shear failure (Figure 4-6) may exhibit both general and punching shear characteristics, soil compression beneath the footing, and possible ground surface bulging.

Refer to Figure 4-7 for photographs of actual test failures using a small steel rectangular plate (about 6 inches wide) and sand of different densities.

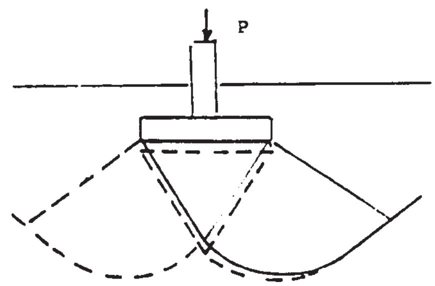


Figure 4-6: Local Shear Failure

Punching shear failure pattern under a rectangular foundation on the surface of loose sand (D_r = 15%). (From De Beer and Vesic, 1958.)

Local shear failure pattern under a rectangular footing on medium dense sand (D_r = 47%). (From De Beer and Vesic, 1958.)

General shear failure pattern under a rectangular footing on dense sand (D_r = 100%). (From De Beer and Vesic, 1958.)

Figure 4-7: Failure Modes

The failure mode to be expected for a given soil profile cannot be predicted. The statement can be made, however, that the mode of failure depends substantially on the compressibility or incompressibility of the soil mass. This is not to imply that soil type alone determines failure mode. For example, a shallow footing supported on a very dense sand will usually fail in general shear, but the same footing supported on a very dense sand which is underlain by a soft clay layer may fail in punching shear.

The ultimate bearing capacity of a given soil mass under spread footings for permanent construction is usually determined by one of the variations of the general bearing capacity equation which was derived by Terzaghi and later modified by Mererhof. It can be used to compute the ultimate bearing capacity as follows:

$$q_{ult} = \frac{\gamma B}{2} N_\gamma + cN_c + \gamma D_f N_q \quad \text{(Terzaghi)}$$

where: q_{ult} = ultimate bearing capacity

γ = soil unit weight

B = foundation width

D_f = depth to the bottom of the footing below final grade

c = soil cohesion, which for the undrained condition equals:

$$c = s = \frac{1}{2} q_u$$

where: s = soil shear strength

q_u= the unconfined compressive strength

In the above equation, N_γ, N_c, and N_q are dimensionless bearing capacity factors that are functions of the angle of internal friction. The term containing factor N_γ shows the influence of soil weight and foundation width, that of N_c shows the influence of the soil cohesion, and that of N_q shows the influence of the surcharge.

Factors Affecting Bearing Capacity

When the supporting soil is a cohesionless material, the most important soil characteristic in determining the bearing capacity is the relative density of the material. An increase in the relative density is accompanied by an increase in the bearing capacity. Relative density is a function of both ø and γ, the angle of internal friction and unit weight, respectively.

For cohesive soils, the unconfined compressive strength q_u, which is a function of clay consistency, is the soil characteristic affecting bearing capacity. The bearing capacity increases with an increase in q_u values.

The location of the water table surface is another factor having a significant impact on the bearing capacity of both sand and clay.

When the depth of the water table from the bottom of the footing is greater than or equal to the width of the footing B, the full soil unit weight is used in the general bearing capacity formula. At these depths, the bearing capacity is only marginally affected by the presence of water and can therefore be neglected. When the water table is at or below the base of the footing, the submerged unit weight, $\gamma_{sub} = \gamma_{sat} - \gamma_w$, is used in the first term of the bearing capacity equation. The result, when the water table is at the bottom of the footing, is to reduce the first term of the equation by approximately 50%. If the water table is above the bottom of the foundation, the surcharge unit weight is also affected, and the submerged unit weight must be used in the third term of the equation.

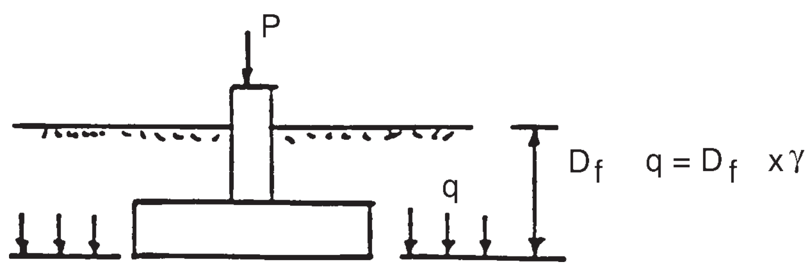


Figure 4-8: Surcharge Load on Soil

Moisture reduces the apparent cohesion of clay and therefore, the shear strength. The unit weight of clay is also reduced when submerged in water (saturated). In saturated clays in undrained shear, the foundation width has little effect on bearing capacity.

It is apparent that bearing capacity of both cohesionless and cohesive soils will be reduced by rising water tables. This can be seen in the general bearing capacity formula when the lighter submerged unit weight of soil is substituted for the dry unit weight. Therefore, the effects of a rising water table on the bearing capacity of the footing soil mass, at any time during construction, must be considered.

Structure related factors affecting the bearing capacity are the depth of footing below ground line (D_f) and the footing shape.

The term D_f is used in determining the overburden, or surcharge, load acting on the soil at the plane of the bottom of footing (Figure 4-8). This surcharge load has the net effect of increasing the bearing capacity of the soil by restraining the vertical movement of the soil outside the footing limits.

Theoretical solutions for ultimate bearing capacity are limited to continuous footings (LENGTH/WIDTH $\geq$ 10). Shape factors for footings other than continuous footings have been determined primarily through semi-empirical methods. In general, the ultimate bearing capacity of a foundation material supporting a square or rectangular footing is greater than for a continuous footing when the supporting material is clay and less when the supporting material is sand.

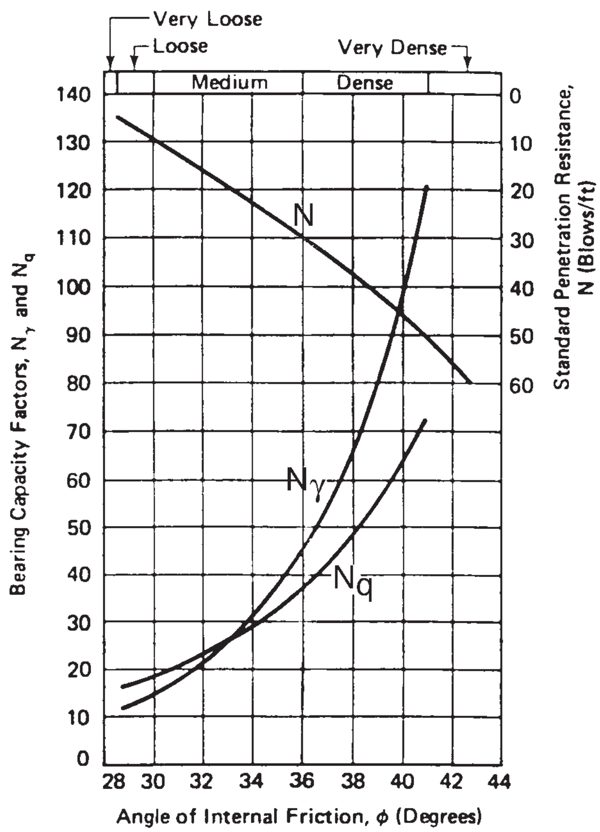


Figure 4-9: Bearing Capacity Factors for Granular Soils

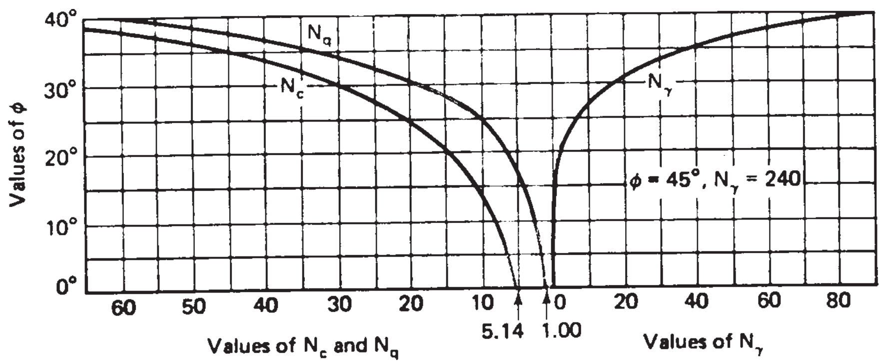


Figure 4-10: Bearing Capacity Factors for Cohesive Soil

The general bearing capacity equation can also be used to give a field estimate of the ultimate bearing capacity of temporary footings, such as falsework pads. For granular soils, a relationship between the standard penetration resistance, N, and the bearing capacity factors, N_γ and N_q, is shown in Figure 4-9. For cohesive soils, the relationship between N and the angle of internal friction, ø, is shown in Figure 4-9. The value of ø determined from Figure 4-9 can then be used to determine the bearing capacity factors, N_γ, N_c, and N_q, as shown in Figure 4-10. Values for ø, q_u, N, and γ can be approximately determined using the tables for granular and cohesive soils shown in Appendix A. Values for N can also be determined from the Log of Test Borings.

Settlement

The allowable bearing capacity for granular soils will almost always be governed by tolerable settlement. When the Structure Foundations Branch of the Office of Structural Foundations recommends spread footings for bridge structures, they specify net allowable bearing pressures that will produce no more than $^1/_2$ inch of maximum settlement or $^1/_2$ inch of differential settlement between footings. These allowable bearing pressures are generally 25% to 33% of the ultimate bearing capacity as determined by the general bearing capacity formula.

Frequently bridge fills are constructed on unstable foundation material. When this occurs, the Structure Foundations Branch will specify that the foundation area be pre-loaded with a surcharge for a specified length of time, known as a settlement period, prior to the start of structure construction. The loading consists of an embankment constructed to specified limits. The Structure Foundations Branch will determine the need to pre-load the foundation area, specify the limits of the embankment, and set forth the duration of the settlement period in the contract Special Provisions.

When settlement periods in excess of 30 days are specified, settlement platforms will usually be required. The Foundations Testing and Instrumentation Section will furnish and supervise the installation of the settlement platforms (Refer to Appendix C for California Test 112 - Method for Installation and Use of Embankment Settlement Devices). A change order written by the Engineer to compensate the Contractor for the initial installation of the settlement platforms will be required.

For settlement periods of less than 30 days, the Structure Representative may install settlement hubs in the top of the bridge embankments. The Structure Representative can then monitor and record the hub elevations weekly. Because the Structure Representative is responsible for terminating a settlement period, data from the hub elevations can be used to determine when this should take place. At the end of the 30 day settlement period, if settlement is still taking place, the settlement period should be extended until settlement has ceased. However, if no settlement has occurred during the last week or two of the settlement period, the Structure Representative should terminate the settlement period at the end of the 30 day period or may alternatively elect to shorten the length of the settlement period. The Contractor should be notified of this decision in writing.

Construction and Inspection

In order to anticipate possible future foundation problems and then formulate some possible solutions before construction begins, the Structure Representative should review and have a complete understanding of all contract documents. As soon as practical, the Contractor should also be reminded that footing elevations shown on the plans are approximate only and foundation modifications are possible. The Structure Representative should draft a letter reminding the Contractor of the provisions stated in Section 51-1.03 of the *Standard Specifications* (refer to Appendix C for sample letter).

The Structure Representative should then review the following documents:

Contract Plans. . .Check footing elevations and adequate cover, design bearing pressures, special treatment of foundations, proximity of utilities, existing structures, highways and railroads, etc.

Special Provisions...Review sections on earthwork, concrete structures, order of work, etc.

Log of Test Borings...Check soil profile, groundwater, etc.

Foundation Report

Standard Specifications...Review the appropriate sections of the *Standard Specifications* relating to construction methods for spread footings.

Section 19-6.01: When bridge footings are constructed in embankment, the embankment shall be constructed to the elevation of the grading plane and the finished slope extended to the grading plane before excavating for the footings.

Section 19-6.025: When a surcharge and settlement period are specified in the *Special Provisions*, the embankment shall remain in place for the required period before excavating for footings. Also defines the minimum limits of embankment that must be constructed before the settlement period can begin.

Section 51-1.03: Plan footing elevations are considered approximate only and the Engineer may order changes in dimensions and/or elevations of footings as may be necessary to obtain a satisfactory footing. The Contractor is responsible for costs incurred due to fabrication of materials or other work prior to final determination of footing elevations. The Contractor should be notified in writing of the possibility of foundation changes prior to commencing foundation excavation operations (refer to Bridge Construction Memo 2-9.0).

Section 19-3.07: When the Engineer determines that it is necessary to increase the depth or width of the footing beyond that which is shown on the plans, for a depth of up to 2 feet below the planned footing elevation or for a width of up to 3 times the planned footing width, increased structure excavation quantities will be paid for at the contract price per cubic yard for structure excavation.

Section 19-3.05: The Contractor shall notify the Engineer when the footing excavation is substantially complete and is ready for inspection. No concrete shall be placed until the footing has been approved by the Engineer.

Section 19-5.03: Relative Compaction of not less than 95% is required for embankments under the bridge or retaining wall footings not supported on piles.

Section 19-3.04: Discusses acceptable methods for removing water from excavations where seal course concrete is specified (or not specified). For footings supported on an excavated surface other than rock, suitable foundation material encountered at the planned footing elevation which has been disturbed or removed by the Contractor shall be restored by the Contractor, at the Contractor's expense, to a condition at least equal to the undisturbed foundation, as determined by the Engineer. If groundwater is encountered during excavation, dewatering shall be commenced and shall precede in advance of or concurrent with further excavation. When unsuitable foundation material is encountered at planned footing elevation, the corrective work will be as directed by the Engineer. Payment for additional work will be at contract prices (preferred) and/or extra work, as determined by the Engineer. When footing concrete or masonry is to rest upon rock, removal shall be as required to expose sound rock. Bearing surfaces shall be roughly leveled or cut to steps and roughened. Seams shall be pressure-grouted or treated as directed by the Engineer and the cost of such work will be paid for as extra work.

Section 51-1.04: Pumping from foundation enclosures shall be done in such a manner as to preclude removal of any portion of concrete materials. Pumping is not permitted during concrete placement, or for 24 hours thereafter, unless it is done from a suitable sump separated from the concrete work.

Section 51-1.09: After placing, vibrating, and screeding concrete in footings that have both a top mat of rebar and are over 2-$^1/_2$ feet deep, the top one foot of concrete shall be reconsolidated as late as the concrete will respond to vibration, but no sooner than 15 minutes after the initial screeding.

Review the appropriate sections of the *Standard Specifications* relating to forms, rebar, concrete, etc..

Section 51: Concrete Structures

Section 52: Reinforcement

Section 90: Portland Cement Concrete

Excavations

An obvious hazard associated with footing foundations is the open excavation. Worker safety must be provided for during excavation operations and/or shoring construction. The Division of Occupational Safety and Health (DOSH) requires that each employee in an excavation be protected from cave-ins by an adequate protective system. The protective system can consist of either metal or timber shoring, a shield system, or a sloping and benching system. When a sloping and/or benching system is substituted for shoring or other protective systems, and the excavation is less than 20 feet deep, DOSH requirements can be selected by the Contractor in accordance with the requirements of Section 1541.1(b) of the Construction Safety Orders. Section 1541.1(b)(1) allows slopes to be constructed (without first classifying the soil) in accordance with the requirements for a Type C soil

(1½:1 maximum). Section 1541.1(b)(2) requires the Contractor's "competent person" to first classify the soil as either a Type A, B, or C soil or stable rock, before selecting the appropriate slope configuration (refer to the Caltrans *Trenching and Shoring Manual* for DOSH Standards when reviewing a Contractor's excavation safety plan).

Surcharge loads must be located a sufficient distance back from the edge of excavations to maintain slope stability. For sloped excavations, the minimum setback can be determined from Figure 4-11.

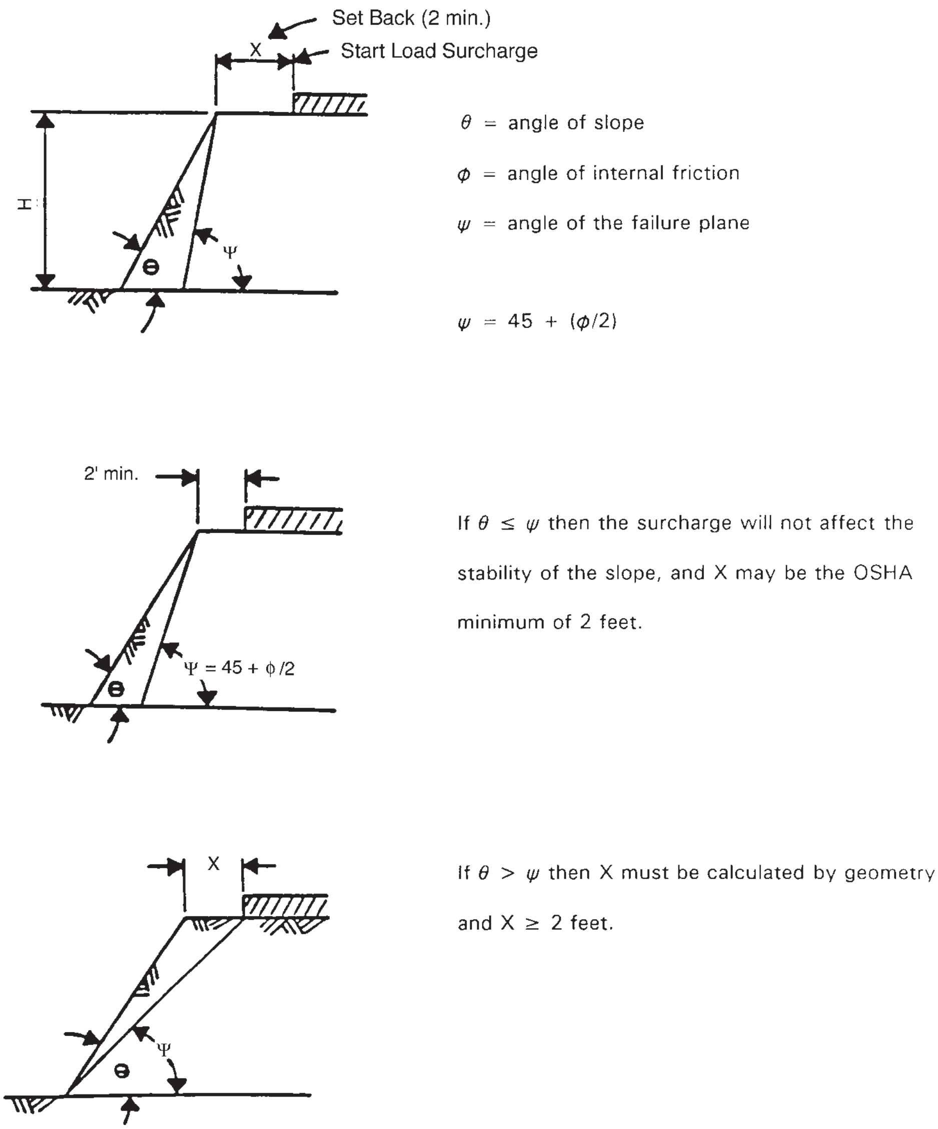
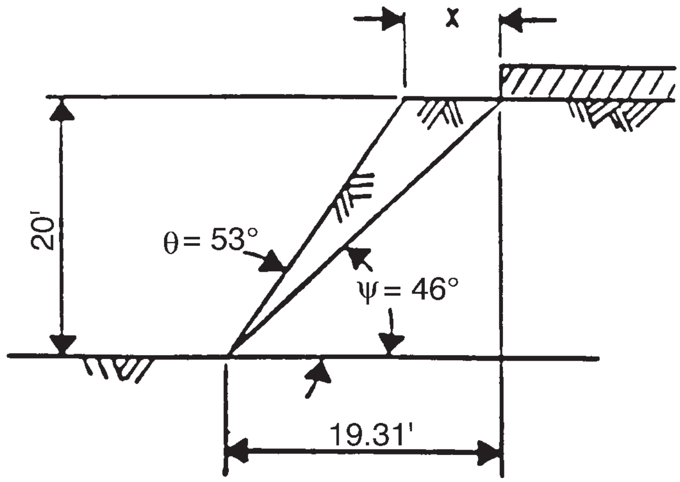


Figure 4-11: Slope Setback

Example:



H = 20'

θ = 53° ($^3/_4$:1 maximum slope allowed by OSHA)

ψ = 46° (Material has little internal friction)

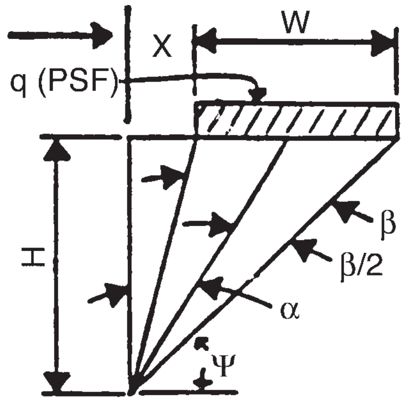
20'/tan 46° = 19.31'

20'/tan 53° = 15.07'

X = 20' - 15.07' = 4.24'

For shored excavations, the minimum setback in level ground is equal to the depth of excavation unless surcharge loads are considered in the shoring design (Figure 4-12).

No setback of the surcharge load is required if the earth support system is designed for the summation of lateral pressures due to the surcharge and earth pressures and a barrier with a minimum height of 18 inches is provided to prevent any debris or other material from entering the excavation.



To calculate lateral pressures due to surcharge, the "Bousineaq" strip load formula is recommended.

At Depth "H",

$$\sigma_h(\text{PSF}) = \frac{2q}{\pi}(\beta_r - \sin\beta\cos 2\alpha)$$

where β_r is in radians

At full height H,

$$\alpha + \frac{\beta}{2} \leq \psi$$

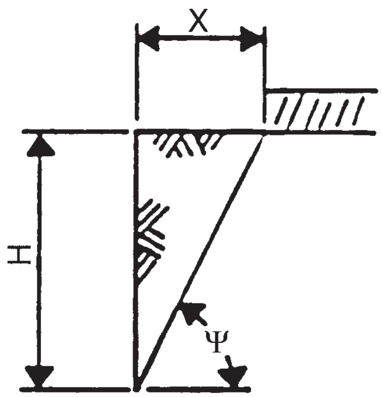
$$\alpha = \arctan\frac{X}{H} + \frac{\beta}{2}$$

$$\beta = \arctan\frac{X+W}{H} - \arctan\frac{X}{H}$$

$$W_{max} = \tan\psi H - \tan\left(\arctan\frac{X}{H}\right)H$$

Figure 4-12: Surcharge Loads for Shored Excavations

If the earth support system is not designed for lateral pressures due to surcharge, then a setback distance must be used, which is calculated as shown in Figure 4-13.



Let X = setback of surcharge load

$$X = \frac{H}{\tan\left(45 + \frac{\phi}{2}\right)}$$

$$\phi = \angle \text{ of internal friction}$$

$$\psi = \angle \text{ of failure plane} = 45 + \frac{\phi}{2}$$

For most soils, ψ is about 55°

Figure 4-13: Setback Calculation for Shored Excavations

Refer to the Caltrans *Trenching and Shoring Manual* for information regarding shoring design and construction.

Wet Excavations

Sump pumps are frequently used to remove surface water and a small infiltration of groundwater.

Sumps and connecting interceptor ditches should be located well outside the footing area and below the bottom of footing so the groundwater is not allowed to disturb the foundation bearing surface.

In granular soils, it is important that the fine particles not be carried away by pumping. Loss of fines may impair the bearing capacity and cause settlement of existing structures. The amount of soil particles carried away can be determined by periodically collecting discharge water in a container and observing the amount of sediment.

If there is a large flow of groundwater and prolonged pumping is required, the sump(s) should be lined with a filter material to prevent or minimize loss of fines.

When it becomes necessary to lower the water table, one commonly used method is the single well point system (Figure 4-14).

A well point is a section of perforated pipe 2 to 3 inches in diameter and 2 to 4 feet in length. The perforations are covered with a screen and the end of the pipe is equipped with a driving head and/or holes for jetting. Well points are connected to 2 to 3 inch diameter riser pipes and are inserted into the ground by driving and/or jetting. The riser pipes, which are spaced at 2 to 5 foot centers, are connected to a header pipe which is connected to a pump.

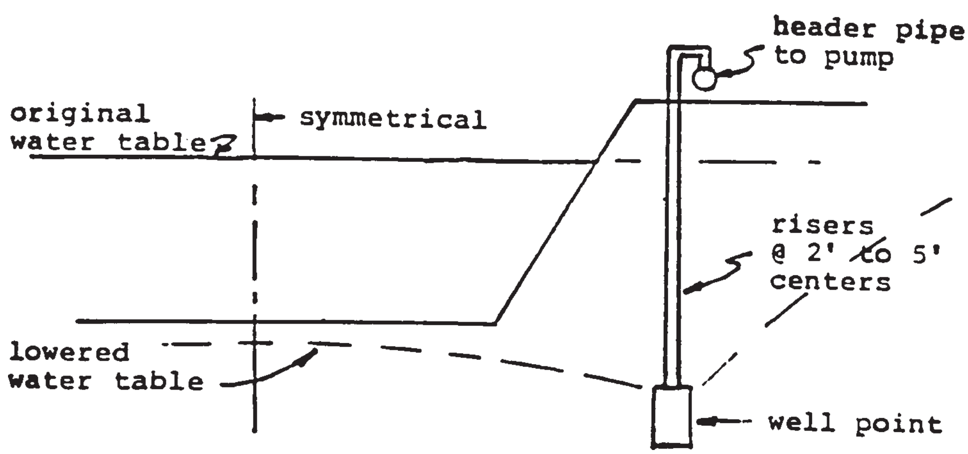


Figure 4-14: Single Stage Well Point System

A single stage well point system can lower the water table 15 to 18 feet below the elevation of the header pipe. For greater depths a multiple stage system must be used.

A single or multiple stage well point system is effective in fine to medium granular soils or soils containing seams of such material. In stratified clay soils, vertical sand drains (auger holes backfilled with sand) may be required to draw water down from above the well points.

Another system for lowering the water table is a deep well. Deep wells consist of either a submersible pump, turbine or water ejector at the bottom of 6 to 24 inch diameter casings, either slotted or perforated. The units are screened but filter material should be provided in the well to prevent clogging and loss of fines.

Deep wells are spaced 25 to 120 feet apart and are capable of lowering a large head of water. They can be located a considerable distance from the excavation and are less expensive than the multiple stage well point system for dewatering large areas.

If a soft clay strata overlying sand is encountered and dewatering is contemplated, the Structure Representative is cautioned that lowering the water table by pumping from underlying layers of sand may cause large progressive settlement of the clay strata in the surrounding area. This is due to consolidation of the saturated clay below the lowered water table caused by an increase in the effective pressure acting on the saturated clay, i.e., density of clay above the lowered water table will increase from a submerged unit weight to a saturated unit weight, an increase of 62.4 Pounds per Cubic Foot (PCF) (Figure 4-15).

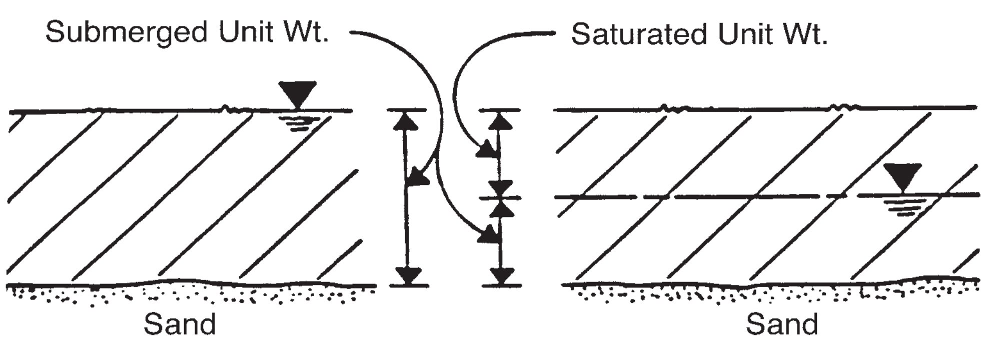


Figure 4-15: Saturated vs. Submerged Unit Weight

Bottom of Excavation Stability

Heave and piping are problems associated with bottom of excavation stability.

Heave is the phenomena whereby the "head" of the surrounding material causes the upward movement of the material in the bottom of the excavation with a corresponding settlement of the surrounding material. Heave generally occurs in soft clays when the hydrostatic head, 62.4(h + z), is greater than the weight of the overburden at the bottom of the excavation, γz (Figure 4-16).

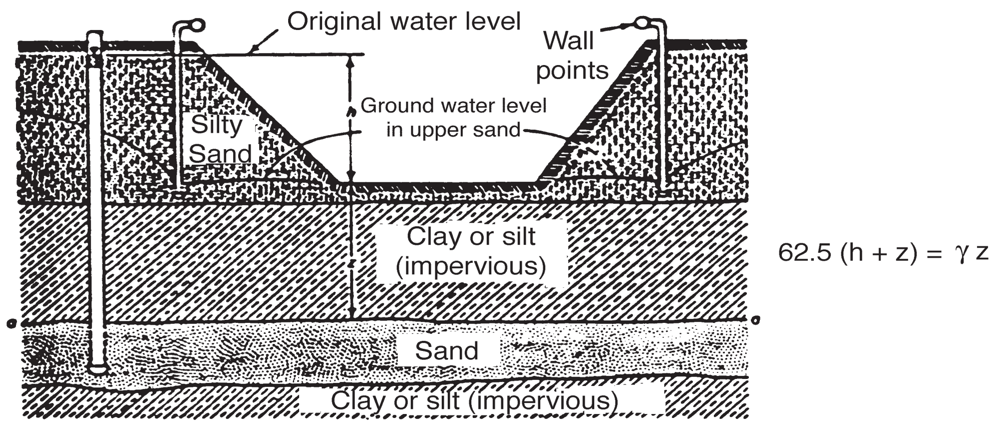


Figure 4-16: Bottom of Excavation Stability Problems due to Excess Hydrostatic Head Against an Impervious Layer

Piping is generally associated with pervious materials and can occur when an unbalanced hydrostatic head exists. This unbalanced head may cause large upward flows of water into the excavation, transporting material in the process, and may result in settlement of the surrounding area. Review the Caltrans *Trenching and Shoring Manual* if instability problems are suspected at the bottom of excavations.

Foundation Inspection

Inspection should include determination of the following:

1) Stability of slopes and sides of excavations conform with DOSH requirements.
2) Conformity of the foundation material with the Log of Test Borings (allowance should

be made for some non-uniformity such as small pockets and lenses of material having somewhat different properties).

3) Condition of the foundation bearing surface (undisturbed by excavation operations and uncontaminated by sloughing and/or entrance of water).

4) Proximity of structures, highways, railroads, and other facilities which may require shoring or underpinning.

5) Forms conform with layout, depth, dimensions, and pour grade of plans. Forms are mortar tight.

6) Reinforcing steel firmly and securely tied in place, shear steel hooked to both top and bottom rebar mats and securely tied. Proper concrete cover over top rebar mat (2 inch minimum rebar clearance to wood forms, 3 inch minimum for neat formed excavation).

7) Concrete placing operations: proper mix number, truck revolutions, back-up alarm, concrete temperature. Wet down rebar and forms, do not allow concrete to drop over 8 feet, reconsolidate and finish top one foot of concrete no sooner than 15 minutes after initial screeding, then cure.

Footing forms are either built-up or consist of prefabricated panels. The forms are generally secured at the bottom by stakes, horizontal kickers or ties and are externally braced, tied or strapped at the top. If the forms extend above the top of footing elevation, a pour strip or similar device must be attached to the forms to designate the top of footing elevation.

Often, the footings are excavated "neat," (excavated to the footing dimensions) and the concrete placed against the sides of the excavation, eliminating the need for footing forms. Top of footing grades must be clearly delineated with stakes or flagged spikes driven into the sides of the excavation. A bench of sufficient width to prevent sloughing or cave-in should be provided around the excavation for access and work area. Ensure that "neat" excavations conform to the planned footing dimensions, or if not, place the exact footing dimensions as constructed on the "As-Built" drawings. On recent seismic retrofit projects, several costly Contract Change Orders have had to be written to correct past undocumented footing overpours.

Whether footings are formed or excavated "neat", a template should be constructed for positioning vertical reinforcing steel cast in the footing to prevent displacement of the vertical rebar during the pour.

All reinforcing steel must be securely blocked and tied to prevent vertical and/or lateral displacement during concrete placement. Reinforcing steel should not be hung or suspended from the formwork or templates (the rebar weight can cause settlement in the form panels affecting pour grades). Top reinforcing steel mats supported by chairs should be blocked to the forms or sides of the excavation. The bottom reinforcing steel mat which supports the vertical column steel should be blocked to prevent any settlement. It is required that all reinforcing steel dowels be tied in place prior to concrete placement and not "stuck in" during or after concrete placement.

The effective depth of reinforcing steel is critical and must always be checked. For a footing supporting a single column, pier or wall, the effective depth is the distance from the centroid of the reinforcing steel to the top of the concrete footing. The bottom mat should be located at the design depth, even for over-excavated footings, since the vertical column reinforcement is supported by the bottom mat and the location of the top mat is tied to the bottom mat by the shear hooks. Lowering the bottom mat is not desirable as it would require longer vertical steel, longer shear hooks, and may require welded splices on the longitudinal bars. It should be noted that the additional concrete placed below the bottom steel mat in over-excavated footings does not increase the design depth of the footing.

Immediately prior to placing concrete, all material which has sloughed into the excavation must be removed. Check again the clearance between the bottom of the excavation and the bottom reinforcing steel mat. The foundation material should be wet down but not saturated. To avoid segregation of the concrete, the ends of the concrete pour chutes should be equipped with a hopper and length of tremie tube to prevent free fall of concrete in excess of 8 feet.

Foundation Problems and Solutions

It is mandatory that the Engineer inspect the excavated surface at the planned footing elevation after the excavation is completed (Section 19-3.05 of the *Standard Specifications* requires the Contractor to notify the Engineer after the excavation is completed). Only by visual inspection can the Engineer determine if the foundation material is suitable, disturbed and/or contaminated, or unsuitable.

Suitable Foundation Material

If the foundation material encountered at planned footing elevation is suitable, the Contractor should be notified in writing of the Engineer's decision.

Disturbed and/or Contaminated Material

A suitable foundation material encountered at planned footing elevation but disturbed or contaminated is unacceptable and must be corrected. Disturbance of the foundation bearing surface is invariably caused by the Contractor's choice of excavation methods. Often the bearing surface is disturbed simply by excavating below the footing elevation, and occasionally the bearing surface is disturbed at grade by the teeth on the excavator bucket. Contamination is usually due to water or sloughing.

All disturbed or contaminated material must be removed to expose a suitable foundation surface. The foundation shall then be restored by the Contractor, at the Contractor's expense, to a condition at least equal to the undisturbed foundation as determined by the Structure Representative.

Acceptable restoration methods include:

1) Maintain top of footing as planned and overform footing depth. With few exceptions, the Contractor will choose this method when the restoration depth is about one foot or less.
2) Replace the material removed (to planned bottom of footing elevation) with Class C (4 sack) concrete.
3) Footings having a design bearing pressure of not more than 3 TSF and where the depth of the material removed does not exceed one foot, the bottom of footing may be restored with structure backfill material compacted to 95% relative compaction. Structure backfill material must meet the requirements of Section 19-3.06 of the *Standard Specifications*.

It cannot be over-emphasized that the restored foundation must be at least equal to the undisturbed foundation as determined by the Engineer.

It is recommended that the following precautionary measures be taken during excavation and construction in order to avoid or minimize disturbance and/or contamination of the foundation surface:

1) Under-excavate with mechanical equipment and hand excavate to bottom of footing.
2) Divert surface water away from the excavation.
3) Minimize exposure of the foundation material to the elements by constructing footings as soon as possible after excavation.

Unsuitable Foundation Material

The Foundation Investigations Section should be contacted when the Engineer determines that the undisturbed original material encountered at planned footing elevation is either unsuitable or of a questionable nature.

If the Structure Representative is absolutely certain that the material encountered at the planned footing elevation is unsuitable, then hand-excavating a small exploratory hole would be advisable prior to contacting the Engineering Geologist. First, review the Log of Test Borings to determine if the unsuitable material is shown but at a higher elevation. If so, the anticipated suitable material may well be just below the excavated surface.

Simple tests that can be performed in the field are:

1) Penetration tests - granular soils
2) Finger tests - cohesive soils
3) Pocket penetrometer - cohesive soils
4) Rodding and probing

Note that these simple and expeditious tests give only an approximate evaluation of the soil at or immediately below the surface. If the bearing capacity of the foundation material is questionable for any reason, consult the Engineering Geologist.

The depth of any hand excavation should not exceed 2 feet, however, because lowering a footing 2 feet or less is not considered a change in the plans or specifications. In any event, the information obtained from the exploratory excavation will be useful in determining the required footing modifications. The required footing modifications will be determined by the Engineer in consultation with the Engineering Geologist, the Bridge Construction Engineer, and the Project Designer.

Footing modifications normally entail one or more of the following procedures:

1) Excavate to a stratum that has sufficient bearing capacity, replace the removed unsuitable material with Class C concrete, and then construct the footing at the planned footing elevation.
2) If the over-excavation is relatively shallow, about one foot or so, replace the removed unsuitable material with footing concrete placed monolithically with the footing.
3) Lower the footing to a stratum that has sufficient bearing capacity and increase the height of the column or wall. This method may not be acceptable if the increase in height necessitates redesign of the column or wall.
4) Increase the footing size so that the bearing pressure does not exceed the allowable bearing capacity of the foundation material encountered at the planned footing elevation. In addition, settlement must not exceed tolerable limits.

Although footing revisions are contemplated by the contract documents, footing revisions made necessary due to unsuitable material encountered at the planned footing elevation will require a change order.

The preferred method for compensating the Contractor for the cost of the corrective work is by contract items at contract unit prices and is the specified method of payment for the following revisions:

1) Raising the bottom of a spread footing above the elevation shown on the plans.
2) Lowering the bottom of a spread footing 2 feet or less below the elevation shown on the plans.
3) Increasing or decreasing the thickness, or elimination of the entire seal course.

For other revisions, agreed price or force account methods should be used when the above method is unsatisfactory as determined by the Engineer.

Safety

Any excavation in which there is a potential hazard of cave-in or moving ground requires a protective earth retaining plan. Section 5-1.02 of the *Standard Specifications* requires the Contractor to furnish a temporary earth retaining system plan to the Engineer for approval prior to starting excavation. Also prior to beginning any excavation 5 feet or more in depth into which a person is required to descend, the Contractor must first obtain a DOSH excavation permit.

Regardless of the worker protection system used, the Contractor's Shoring Plan or Excavation Safety Plan should be inspected to ensure compliance with DOSH requirements.

Daily inspections (or after any hazard-increasing occurrence) of excavations or protective systems shall be made by the Contractor's "competent person" for evidence of any condition that could result in cave-ins, failure of a protective system, hazardous atmospheres, or any other hazardous condition. When any evidence of a situation is found that could result in a hazardous condition, exposed employees shall be removed until the necessary precautions have been taken to ensure their safety.

Safety railing must be located at the excavation perimeter, preferably attached to the shoring that extends above the surrounding ground surface. If the shoring does not extend above the ground, then the railing must be located a sufficient distance back from the excavation lip to adequately protect workmen in the excavation from being injured by falling objects or debris. Locating the safety rail back away from the excavation lip usually provides more stable ground to anchor the rail posts. Spoil piles must be located more than 2 feet away from the excavation lip for excavations deeper than 5 feet.

Although the vertical side of a non-shored excavation must be less than 5 feet in height, care must be exercised when working around the perimeter to avoid falling into the excavation because of sloughing or slip-out of the material at the excavation lip. Spoil piles must be located at least one foot away from the excavation lip for trenches less than 5 feet in depth.

Whenever work is proceeding adjacent to or above the level of vertical projections of exposed rebar, workers shall be protected against the hazards of impalement on the exposed ends of the rebar. The impalement hazard can be eliminated by either bending over the ends of the projecting rebar, or by use of one of the following methods:

1) When work is proceeding at the same level as the exposed protruding rebar, worker protection can be provided by guarding the exposed ends of rebar with DOSH-approved protective covers, troughs, or caps. Approved manufactured covers, troughs, or caps will have the manufacturer's name, model number, and the Cal/OSHA approval number embossed or stenciled on the cover, trough, or cap. Any manufactured protective device not so identified is illegal.
2) When work is proceeding above any surface of protruding rebar, impalement protection shall be provided by the use of: (1) guardrails, (2) an approved fall protection system, or (3) approved protective covers or troughs. Caps are prohibited for use as impalement protection for workers working above the level of the protruding rebar.

Protective covers used for the protection of employees working above grade shall have a minimum 4 x 4 inch square surface area. Protective covers or troughs may be job-built, provided they are designed to Cal/OSHA minimum standards, that the design of the cover or trough was prepared by an Engineer currently registered in the State of California, and a copy of the Cal/OSHA approved design is on file in the job records prior to their use.